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# EARTHQUAKE PERFORMANCE EVALUATION OF A TYPICAL BRIDGE STRUCTURE DESIGNED BY FORCE-BASED DESIGN METHOD IN PHILIPPINES

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## *Abstract*

The government of the Philippines defines the Bridge Seismic Design Specifications (BSDS) to ensure the safety of bridges to strong earthquake ground motions. The last revision year is 2013. The BSDS adopts the concept of the force-based design (FBD). According to this concept, structural engineers estimate linear lateral forces based on the design response acceleration spectrum specified for each seismic zone and each soil type at first. And they verify the strength capacities of the bridge surpass the lateral design forces which are determined by dividing the linear lateral forces by the response modification factor (R-factor). The R-factor represents the ductility of structure. The BSDS defined these factors according to the structural characteristics of bridges such as the number of piers. Although appropriate R-factors restrain excessive plastic deformation, the engineers cannot predict it precisely by the FBD. Therefore, lots of engineers currently insist they should introduce the concept of the displacement-based design (DBD) in the view of seismic damage–control design.

In this study, we attempt to apply the DBD to an existing typical bridge in the Philippines referring to the Japan Road Association Specification for Highway Bridges (JRASHB, 2002), and discuss merits of the DBD based on the application results. The target bridge is a prestressed girder bridge with three spans. Each span length is about 20m. The substructure is composed of 2 by 2 RC piers supported by bored bent piles and abutments. This bridge was designed by the previous BSDS. The applied R-factor was 5. After the revision of the BSDS in 2013, the R-factor is specified according to three operational function categories in addition to seven structural types. If the target bridge's function is categorized into 'essential', the R-factor should be reduced from 5 to 3.5. Thus, seismic performance evaluation results of this bridge based on a more precise method is also our concern.

We determined the force-displacement relationship and the limit state displacements for the collapse prevention (Limit state - 2) of our target bridge based on the JRASHB and carried out response history analyses (RHA) using artificial ground motions which fit the design acceleration response spectrum in the Philippines. Furthermore, we applied the capacity spectrum method as a simplified seismic response prediction method instead of the RHA. The results showed that response displacements became lower than the corresponding limit sate displacements. However, the large plastic deformation occurred, and the rotation of the plastic hinge had reached to about 0.02 rad. To improve the safety more, we suggested revising the dimension of cross-section, the arrangement of main steel bars and so on. By these revisions, the plastic hinge rotation was reduced to about 0.01 rad. This revised model also satisfied the current BSDS revised in 2013. By comparing earthquake response displacements with corresponding limit displacements as demonstrated in this study, we can precisely predict damage aspects of the structure and assess its seismic performance more appropriately. We consider this is one of the merits of the DBD.

*Keywords: Displacement-based design, Damage control, Force-based design, Response modification factor, Bridge structure*

## 1. Introduction

The Philippines is situated within the Pacific Ring of Fire which exhibits high seismic activity compared to the other parts of the world. This geographic characteristic results into increased exposure of natural disasters such as earthquakes. In the event of an earthquake disaster, the accessibility of a stricken area is vital not only for the evacuation of victims but also for the rescue and response operations for other casualties. Regarding the connectivity of the transportation network, bridges play an essential role as it can indicate the only means of passage for a community. This is especially the case for rural locations. Past earthquakes in the country resulted into numerous damaged bridges. Large displacements from the columns resulted in the collapse and unseating of the superstructure which directly affects the operation of the bridge thus isolating a community. These structures were evaluated based on the previous specifications for seismic design through the Force-Based Design Method (FBD). This method demonstrates that force is the main criterion in evaluating the demand capacity of the structure. The linear elastic procedure is effective as long as the structure performs within elastic limits. This approach cannot precisely demonstrate the behavior of the component especially when damage starts to occur in the component. This study proposes an alternative method in evaluating the seismic performance for bridges. This method is called the Displacement-Based Design Method (DBD). Under this method, the primary criterion is based on displacement, where damage can be checked better.

## 2. Target bridge and applied design method

### 2.1 Target bridge

The target bridge was designed using the Department of Public Works and Highways'(DPWH) Design Guidelines, Criteria and Standards (DGCS) 2004 Edition stipulated by the Department Order No. 75 Series of 1992. The Bridge Seismic Design Specifications (BSDS) is a part of this DGCS. The DGCS adopts the concepts from the American Association of Highway and Transportation (AASHTO) Standard Specification for Highway Bridges, 19th Edition, 2002. Fig. 1 shows the target bridge.

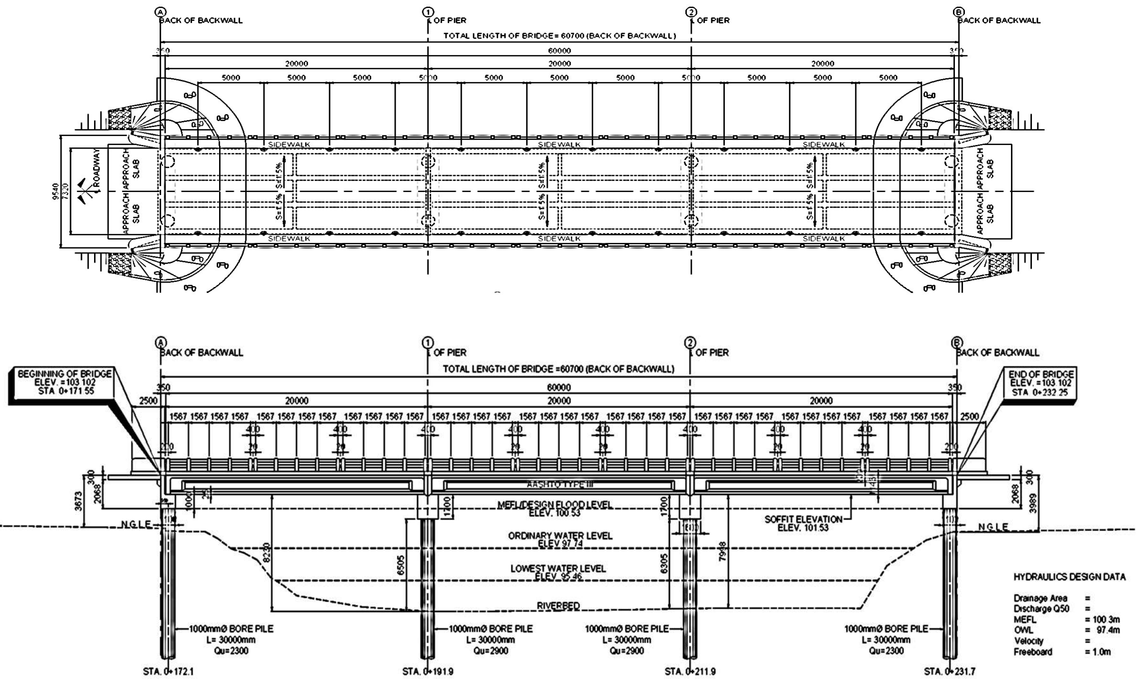



Fig. 1 – General Plan and Elevation of the target bridge



The target bridge considered in this study is a typical bridge structure commonly used in the Philippines. The bridge is a three-span prestressed girder bridge with total length of 60.80 m and total width of 9.34 m. The bridge is supported by two circular columns, which are on a bored pile foundation, and abutments. The total weight of the super structure is 6690 kN. Table 1 shows the material properties. Figure 2 and 3 show a cross section of pier and a pier elevation, respectively. The diameter of the pier is 1.0 m, the unsupported length is 7.35 m, and 9.95 m is the height from the bottom of the pier to the center of the superstructure. The pier is reinforced longitudinally with 18 pieces of 25 mm diameter deformed bars and with lateral reinforcement spirals with 16 mm diameter deformed bars. They are used to confine the concrete core, with a space of 70 mm at the plastic region and 100 mm at the center portion of the pier.

Table 1 – Material properties

| | | |
|---|---|---|
| **Concrete** | | 28 Mpa |
| **Steel** | 16 mm (Hoops) | 275 Mpa |
| | 25 mm (Main Bar) | 414 Mpa |

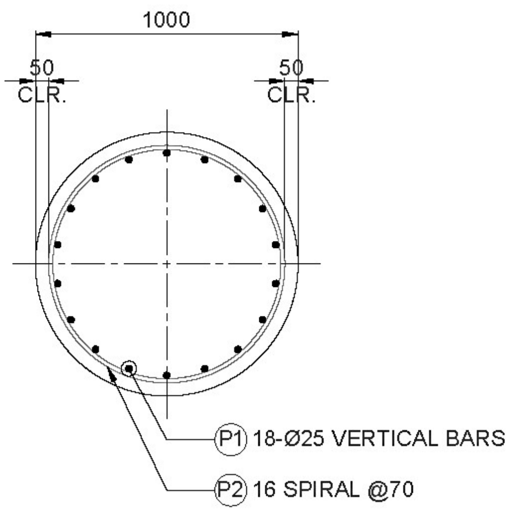



Fig. 2 – Ssection of Pier (plastic region)

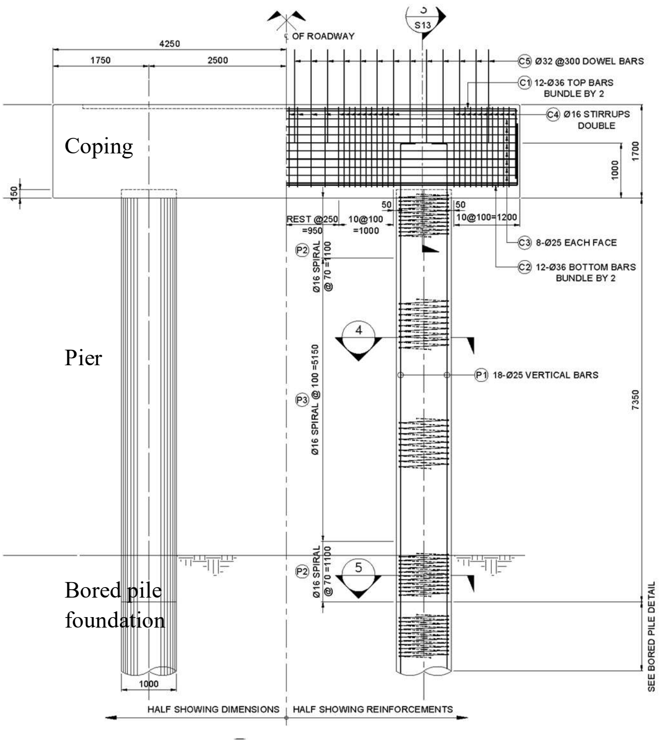



Fig. 3 – Pier elevation

2.2 Design method applied to the target bridge

The force-based design enables us to verify the seismic performance of structures by comparing strength capacities and seismic design forces. Fig. 4 shows the design acceleration response spectrum for the target bridge determined by the seismic code of the Philippines. And, Table 2 shows the principal design coefficients of it. In Fig.4, the first natural periods, which are 1.17 s in the longitudinal direction and 0.14 s in the transverse direction, also presented. The seismic design forces for the force-based design are obtained by dividing the elastic response forces, which are calculated by linear analysis using the design spectrum, with the response modification factor (R-factor.) The R-factor corresponds to ductility of structures. Table 3 shows the R-factors for each structural system regulated in the seismic code of the Philippines before 2013. For the target bridge, the R-factor was regarded as 5. After the revision of seismic code in 2013, the R-factor is defined according to three operational function categories in addition to seven structural types, as shown in Table 4. If the target bridge's function is categorized into 'essential', the R-factor should be reduced from 5 to 3.5.



For the target bridge, the modal analysis method was applied for linear analysis. Natural modes were combined according to the CQC rule. Also, 0.3 times of lateral forces in the orthogonal direction were combined to evaluate effects of bi-directional input ground motions. As in many other cases, commercially available software was used to perform these calculations.

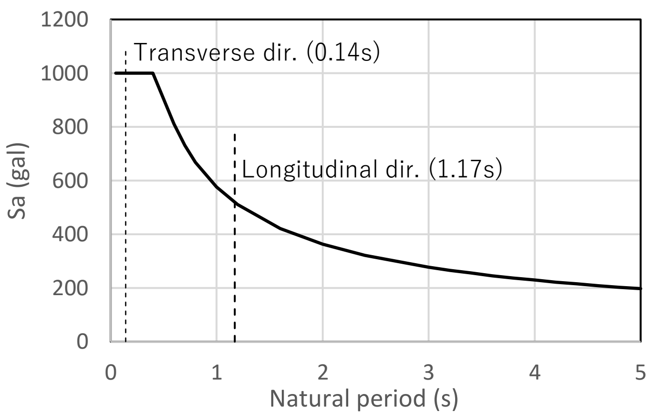


Fig.4 – Design acceleration response spectrum

Table 2 – Principal design coefficients

| | |
|---|---|
| Importance classification | 1.0 |
| Acceleration coefficient | 0.4 (Zone 4) |
| R-factor | 5 |
| Soil classification | 1.2 (Soil type II) |

Table 3 – R-factors in previous code

| **Subtructure**[1] | **R** |
|---|---|
| Wall Type Pier | 2 |
| Reinforced Concrete Pile Bents | |
| a. Vertical Piles only | 3 |
| b. One or more Batter Piles | 2 |
| Single Column | 3 |
| Steel or Composite Steel and Concrete Pile Bents | |
| a. Vertical Piles only | 5 |
| b. One or more Batter Piles | 3 |
| Multiple Column Bent | 5 |

1. The R-factor is to be used for both orthogonal axes of substructure.

Table 4 – R-factors revised in 2013

| **Subtructure**[1] | **Operational Category** | | |
|---|---|---|---|
| | OC-I (Critical) | OC-II (Essential) | OC-III (Others) |
| Wall Type Pier | 1.5 | 1.5 | 2.0 |
| Reinforced Concrete Pile Bents | | | |
| a. Vertical Piles only | 1.5 | 2.0 | 3.0 |
| b. One or more Batter Piles | 1.5 | 1.5 | 2.0 |
| Single Column | 1.5 | 2.0 | 3.0 |
| Steel or Composite Steel and Concrete Pile Bents | | | |
| a. Vertical Piles only | 1.5 | 3.5 | 5.0 |
| b. One or more Batter Piles | 1.5 | 2.0 | 3.0 |
| Multiple Column Bent | 1.5 | 3.5 | 5.0 |



## 3. Seismic Performance Evaluation Considering Deformation Capacities

### 3.1 Outline

In this chapter, we proceed with our study following the steps below.

(1) Evaluation of the deformation capacity of the target bridge

(2) Execution of non-linear history response analyses

(3) Consideration on seismic performance of the target bridge

We attempt to apply the displacement-based design concept for evaluating the seismic performance of the target bridge following the Japan Road Association Specifications for Highway Bridges (2002) [1], which is referred to as JRASHB in the sections below. To simplifying a problem, we deal with only the longitudinal direction and assume the pier bases are fixed, neglecting the flexibility of the soil.

### 3.2 Deformation capacities

For applying the displacement-based design, we need to evaluate deformation capacities of the structure, which we should compare with earthquake displacement responses. To do this, we use the stress–strain relations for concrete and steel formulated by the JRASHB.

Fig. 5 shows the stress-strain relation of confined concrete in the pier shown in Fig. 3 according to the JRASHB. And Fig. 6 shows that of steel bars. As shown in fig 3, hoops are densely arranged in the pier. Thus, we can see an enough effect of confining in the Fig. 5.

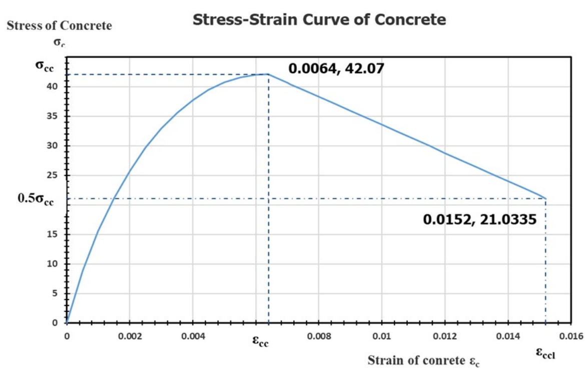



Fig.5 – Stress-strain relation of concrete

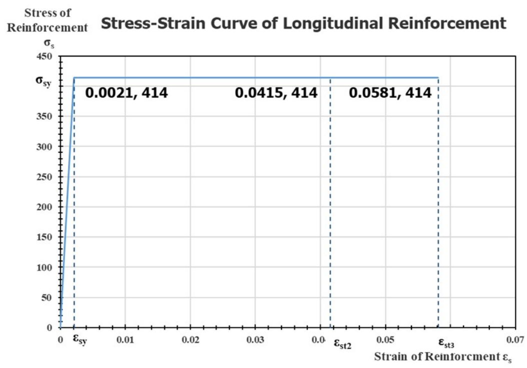



Fig.6 – Stress-strain relation of steel bar

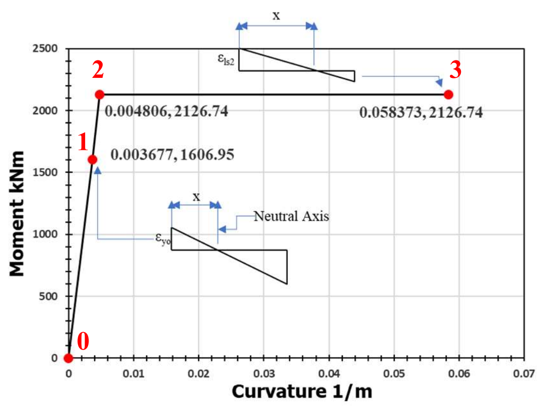



Fig.7 – M–φ relation of pier section

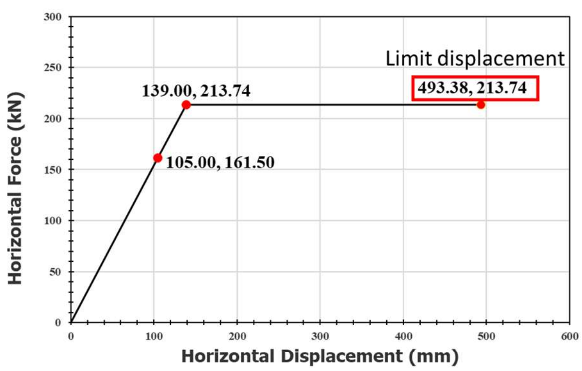



Fig.8 – Lateral force–disp. relation of pier



Fig. 7 shows the moment-curvature relation of the pier, which is applied to increasing moment and constant vertical force. This relation is obtained by the section analysis under the assumption of plane retention. Three results of the section analysis and the original point are presented in Fig. 7. At the point -1 in the figure, the steel bar at the edge starts yielding. The strain reaches to its yielding strain, $\varepsilon_y$. The point -3 reveals the ultimate moment capacity and the limit curvature for the limit state - 2. At this point, the strain of concrete does not reach to its limit shown in the Fig.5. The moment-curvature relation in Fig.7 was simplified to a bi-linear relation. The point -2 is the yield point of this bi-linear relation.

Because we can regard the target bridge in the longitudinal direction as a simple cantilever, we can derive easily the force-displacement relation at the top of the pier as shown in Fig.8. The limit displacement $\delta_{ls2}$ for the limit state - 2 defined in the JRASHB was calculated by Eq. (1).

$$\delta_{ls2} = \delta_y + (\varphi_{ls2} - \varphi_y)\, L_p\, (h - L_p/2) \tag{1}$$

where, $\delta_y$: Yield displacement of the structure calculated by linear analysis, $\varphi_{ls2}$: limit curvature corresponding to the point 3 in Fig. 3, $\varphi_y$: yield curvature corresponding to the point 2 in Fig.3, $h$: height from the bottom to the top of the pier and $L_p$: length of plastic hinge region.

We calculate the length of plastic hinge region $L_p$, with the formula provided by the JRASHB, which evaluates the confining effects by the hoop, and so on. The pier height $h$ of the target bridge is about 10 m. Thus, the limit drift angle is about 0.05 rad. It generally means the target bridge possesses enough ductility. The seismic code regulates some prescriptive details such as minimum requirements of strengths of materials, their qualities, cross-section sizes, and hoop intervals. The code works efficiently to make the bridge ductile enough.

### 3.3 Response history analyses

To predict earthquake response displacements, we conduct response history analyses (RHA). Fig. 9 shows the analysis model for the target bridge. Because we have already obtained the moment-curvature ($M$-$\varphi$) relation, we can apply the elements using the $M$-$\varphi$ relation shown in Fig. 10. This $M$-$\varphi$ relation includes the effects of vertical forces due to the dead and live loads. Thus, we need not estimate the lateral and vertical interaction effects in the numerical model furthermore. However, we apply the fiber element model as an alternative method in this study. Each fiber property is characterized by the stress–strain ($\sigma$-$\varepsilon$) relation. Fig. 11 shows the mesh of the fiber element model. There are two type of fibers. The one is for concrete: the other is for steel. For the hysteresis rule of each fiber, we adopt the Hoshikuma's model for the concrete and the bi-linear model for the steel.

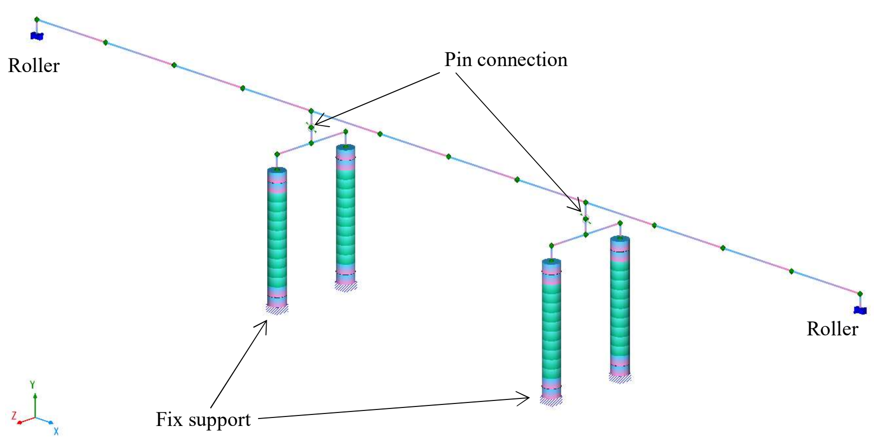



Fig.9 – Analysis model



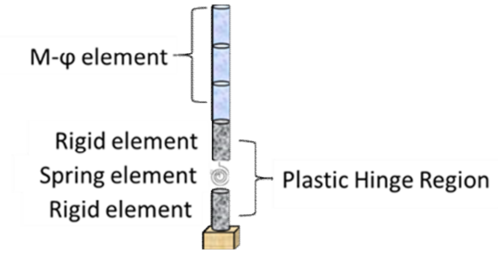


Fig.10 – Simple numerical model

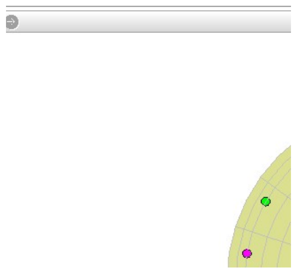
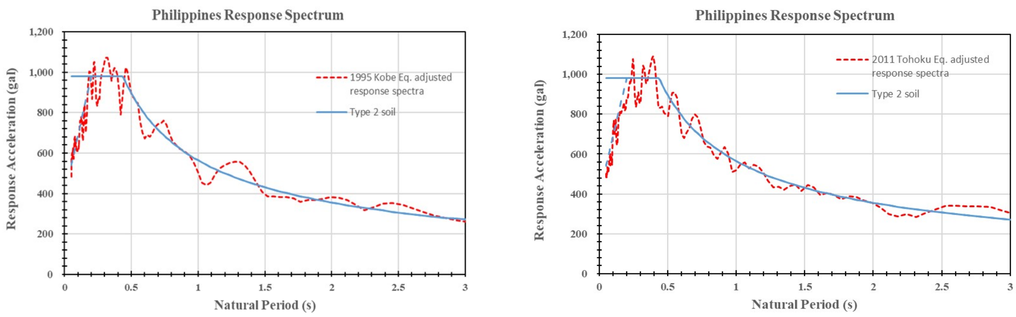


Fig.11 – Mesh of fiber model



(Kobe phase) (Tohoku phase)

Fig.12 – Response acceleration spectrum

We input artificial ground motions to the analysis model for the RHA. We make the acceleration response spectrum of them fit the design spectrum of the seismic code of the Philippines, as shown in Fig. 12 and adopt two series of Fourier phases. The one is those which we abstract from the record measured at the JR Takatori in the 1995 Kobe earthquake: another is from the record at the Sendai River and Highway Office in the 2011 Tohoku earthquake. The ground motion with the Kobe phase represents properties of inland type earthquakes with a short-time duration, which is about 20 s: That with the Tohoku phase represents properties of plate boundary earthquakes with a long-time duration, which is about 300 s.

Table 5 shows the maximum displacements predicted by the RHA, comparing with the corresponding design criteria. The predicted response results fall below the design criteria both in the Kobe and the Tohoku phase.

Table 5 – Verification of maximum displacements

| Displacement (mm) | | Kobe Phase | Tohoku Phase |
|---|---|---|---|
| Yield | $\delta_y$ | 139.00 | 139.00 |
| Limit (State 2) | $\delta_{ls2}$ | 430.38 | 430.38 |
| Max. (Response) | $\delta_{max}$ | 325.34 | 288.00 |
| Remark | | OK | OK |

The JRASHB requires to check the residual displacements which relates to the functionality of the structure after earthquakes, using the following equations.

$$\delta_R \leqq \delta_{RA} \quad (2)$$

$$\delta_R = C_R\,(\mu_r - 1)\,(1 - r) \quad (3)$$



where, $\delta_R$: design residual displacement (mm), $C_R$: modification factor for residual displacement, for RC piers, 0.6, $\mu_r$: maximum response ductility factor of piers, $r$: ration of the secondary stiffness to the initial stiffness of piers, for RC piers, 0.0 and $\delta_{RA}$: allowable residual displacement, 0.01$h$ (mm).

Table 6 shows the verification results by above equations. In the Kobe phase, the predicted residual displacement surpasses the design criterion.

Table 6 – Verification of residual displacements

| Residual Displacement (mm) | | Kobe Phase | Tohoku Phase |
|---|---|---|---|
| Allowable | $\delta_{RA}$ | 99.5 | 99.5 |
| Estimated | $\delta_R$ | 112.0 | 90.7 |
| Remark | | NG | OK |

3.4 Consideration

To understand seismic response characteristics of the target bridge better, we also predicted seismic responses of it by the capacity spectrum method (CSM) [2]. This method enables us to predict the responses by comparing the capacity curve which represents the restoring force characteristics of the structure with the demand spectrum, that is *Sa*-*Sd* spectrum. Figs. 13 and 14 present the application result of this method. The earthquake responses are represented as the intersection of the capacity curve and the demand spectrum. By evaluating an energy dissipation effect by the hysteresis damping of the structure after its yielding, we can reduce the level of the demand spectrum. We evaluate such energy dissipation effect by the equivalent viscus damping ratio *heq* by Eq. (4) and reduce the demand spectrum by reduction factor *Fh* by Eq. (5) [3].

$$heq = 0.25\ (1 - 1/\mu^{0.5}) + 0.05 \quad (4)$$

$$Fh = 1.5\ /\ (1+10\ heq) \quad (5)$$

where, $\mu$: ductility factor.

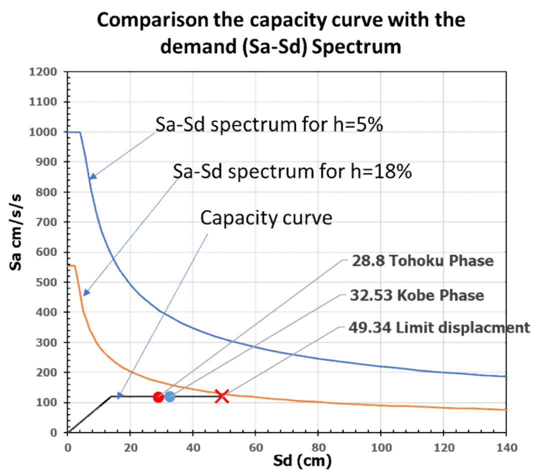



Fig.13 – Results of CSM

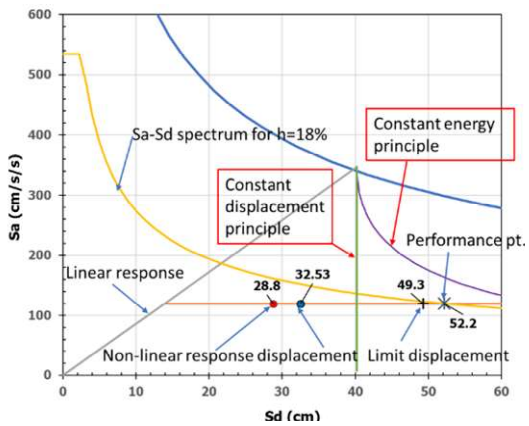



Fig.14 – Expanded figure

Fig.14 shows a part of Fig.13 with expanding it. And in this figure, we present the constant energy principle and the constant displacement principle as a reference. We also plot the results of the RHA in Figs. 13 and 14. The results of the RHA fell below the corresponding limit displacement, 49.3mm, as we already saw in Table 4, too. Contrary, the response displacements predicted by the CSM, which are intersections of the demand spectrum and the capacity curve in Figs. 13 and 14, surpass it. We should note that the CSM could provide conservative predictions because Eqs (4) and (5) include a safety margin. However, from this prediction results of the CSM, we can say that the displacements are likely to be large, and they might exceed the limit even if the seismic force becomes slightly larger than expected. Also, the residual displacement surpassed its criteria for the Kobe phase as shown in the Table 5.

Considering these results, we attempt to revise the section of the piers.

Fig. 15 and 16 show the revised cross-sections. We derive force-displacement relation of each revised model by the same procedure for the original model described in the section 3.2. Tables 7 and 8 show the properties of each model. The force-based design requires the strength ratio Qp/Qe should be less than the R-factor. The elastic force Qe is calculated using the acceleration response spectrum shown in Fig.4. These values for the revised model in Table 7 and 8 are less than the R-factor for the operational category II in Table 4, that is 3.5. It means that the revised models satisfy the strengthened regulation.

Fig.17 shows the prediction results for seismic responses by the CSM. These figures present we can reduce response displacements to about half of that of the original model by revising the cross-section of the pier.

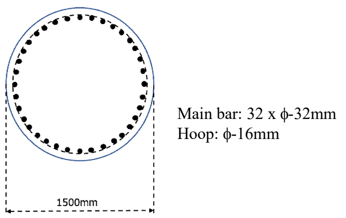



Fig. 15 – Revised cross-section (Case 1)

Table 7 – Properties of revised model (Case 1)

| | |
|---|---|
| Fundamental period | 0.52 s |
| Strength capacity, Qp | 267.3 kN |
| Elastic force, Qe | 504.7 kN |
| Strength ratio, Qe/Qp | 1.9 |

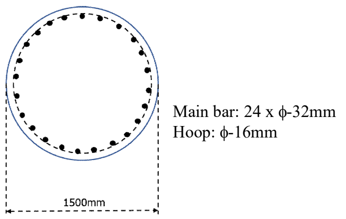
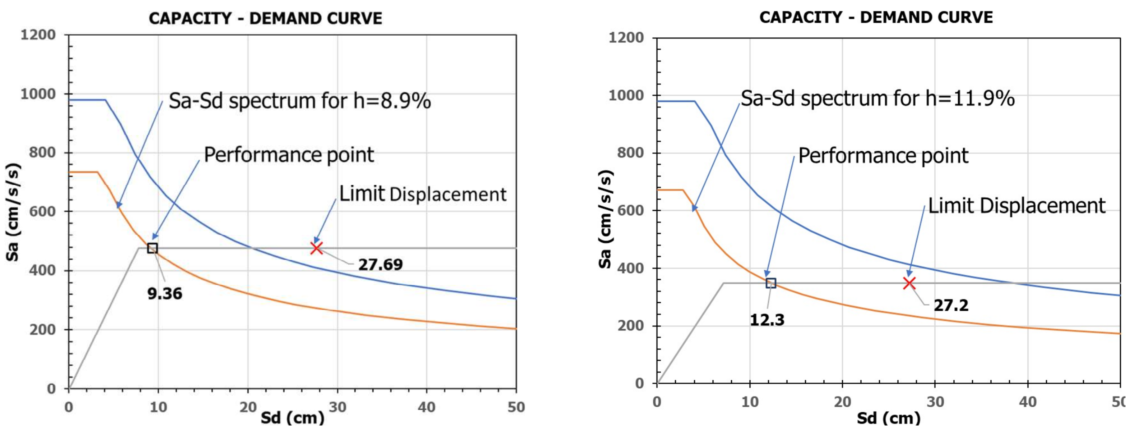



Fig. 16 – Revised cross-section (Case 2)

Table 8 – Properties of revised model (Case 2)

| | |
|---|---|
| Fundamental period | 0.52 s |
| Strength capacity, Qp | 194.8 kN |
| Elastic force, Qe | 504.7 kN |
| Strength ratio, Qe/Qp | 2.6 |




(Case 1) (Case 2)

Fig.17 – Prediction of seismic responses by CSM



## 4. Conclusions

In this study, we attempted to apply the displacement-based method for evaluating the earthquake performance of the typical existing bridge designed by the force-based method. For the displacement-based design, we compare the limit displacements with corresponding earthquake response values. In this study, we determined these limit values by referring to the Japan Road Association Specifications for Highway Bridges, 2002, (JRASHB.) The conclusions are summarized as follows.

1. The results of the response history analyses (RHA) showed that the response displacement did not exceed the limit displacement. It means that the target bridge will not collapse for severe earthquake expected by the seismic code of the Philippines.
2. Although the response displacements predicted by the capacity spectrum method (CSM) corresponded well to those by the RHA, the predicted value exceeded the limit displacement.
3. From the above second result, we considered that it would be better to make the safety of the target bridge higher and proposed the revised cross-sections for the target bridge.
4. The results of the CSM showed the response displacements of the revised bridge models became smaller than the corresponding limit displacement and decreased to half of that of the original model.
5. The revised models also satisfy the current seismic code of the Philippines, which was recently revised in 2013. According to the revised code, the R-factor should be reduced from 5 to 3.5 for the same type bridges as the target if the functional category is the 'Essential'. We confirmed the revised model has higher strength capacity than required one which is determined by the modified R-factor.
6. We should secure the functionality of bridges after earthquakes. Residual displacements correspond to such functionality. We checked the residual displacements of the target bridge according to the JRASHB. The results showed that the residual displacement could exceed the allowable limit under some conditions. This result was also one of the reasons we proposed to revise the cross-section.

We can express the limit states such as immediate occupancy, life safety, collapse, etc. by the corresponding limit displacements. By comparing such limit displacements with an earthquake response displacement, we can precisely predict damage aspects of the structure and assess its seismic performance.

The limit displacements for the structural design reflects specifications and details of each structure which structural engineers adapt flexibly under various construction conditions. Using such parameters, we can evaluate the seismic performance of the structure according to its unique characteristics. This fact may enable us to extend the design freedom.

Also, by focusing on residual displacements of a structure, we can evaluate the functionality or the reparability of the bridge after earthquakes. And, we consider that we need the following items to implement the displacement-based design efficiently.

1) Technical guideline to provide how to evaluate force-deformation relationships of a structure and to determine limit displacements which correspond to some limit states.
2) Practical design computer software, which has functions to evaluate force-deformation relationships from specifications and details of a structure and so on.

## 5. Acknowledgements

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